IDENTIFTYING LOADING AND RESPONSE MECHANISMS FROM TEN YEARS OF PERFORMANCE MONITORING OF A TALL BUILDING

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ABSTRACT

In 1993 Shimizu Corporation provided the opportunity to record manually readings of stress and strain gauges they had embedded at the 18th storey of a 65-storey office tower under construction in Singapore. Static readings continued during construction and long after, and capitalising on access to the building and assistance of both contractor and owner, monitoring systems for tracking wind, acceleration and deflection were installed and progressively upgraded. Further, a comprehensive ambient vibration survey and finite element model updating exercise provided a thoroughly validated analytical model of the structure. This model has been used in parallel with the analog wind and tremor 'super-sensor' of the building itself to provide direct evidence and characterization of the seismic and wind loadings on the building. This paper describes the evolution of the monitoring system and its capabilities together with some of the insights the system provided into structural and loading mechanisms during its operational life until early 2005..

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INTRODUCTION: MOTIVES AND APPLICATIONS FOR STRUCTURAL HEALTH AND PERFORMANCE MONITORING

Structural health monitoring (SHM) for civil infrastructure has many definitions. These include the systems not only for detection/diagnosis of progressive or sudden damage, but also for performance monitoring to identify load/response relationships either as a baseline for the structure itself or for calibration of a loading model or code. The words 'health' and 'performance' are both used in defining SHM systems and are synonymous, and a major activity for civil infrastructure monitoring is geared towards a long term evaluation of what is 'normal' structural performance or health.

As infrastructure owners perceive the need for systems to help them track the performance of their structures, researchers educate them about developments in new tools and techniques and there is a growing overlap of research capabilities with real world needs. The most effective way to accelerate the process is to educate by example of successful implementations; capability is proven and benefits to owners and the community of structural engineers are best presented through case studies.

The monitoring exercise reported in this paper, conducted over the period from late 1993 to early 2005, began life as periodic manual readings of strain gauges. It evolved into a system to provide information about the various forms of environmental loading (wind, tremors and temperature) via measurable responses (stress, strain, acceleration and displacement). Along the way it validated the effectiveness of the design and was used as a test bed and learning experience for implementation of SHM technology.

The paper follows the chronology of the building and monitoring activities, which began with the manual monitoring of embedded static gauges during construction. Bi-directional vibration measurements during visits to read the gauges allowed tracking of natural frequencies as construction progressed. A modal survey of the completed but empty building was followed by permanent installation of a bi-axial acceleration recorder which, with signal cables built into the structure by the contractor, developed into a complete system for recording of wind speed signals together with accelerations from roof and basement levels. The last phase of development was installation of a dual-rover GPS system and integration with the existing systems.

Given the close cooperation with owner, architect and contractor, it was possible to construct a range of representative finite element models of the building and these were fine tuned using a novel form of model updating leading to a reliable simulator for the building to be used for estimating loads by inverse analysis from the response. Computational fluid dynamics and wind tunnel studies were also carried out to characterize the wind flow around and wind loads on the building (Tng at al., 2000).

A number of papers concerning this building have already been published, and the aim of this paper is to present a complete overview of performance. As well as showing long-term trends, some specific results are presented that show how the building was used as a form of wind and seismic 'super-sensor' for direct measurement of seismic ground motions and wind loads.

BUILDING CONFIGURATION

A full description of this 280m, sixty-six storey office tower is provided by the architect elsewhere (Teh & Lai 1995). In summary, the tower has a frame-tube structural system with an internal reinforced concrete core wall connected to a ring of 16 external steel columns by horizontal steel framing system at every floor.

Vertical system

Fig. 1 shows a perspective view of the building and Fig. 2 shows a typical cross section, at storey 18. The structure is built around a reinforced concrete (RC) central core that has almost a square profile with side 22m and which exists for the full height of the building but with varying wall thickness (up to 600mm at lower levels).

The perimeter of the building comprises eight large and eight small steel tube columns over 1m diameter and these connect with the core via a horizontal framing system. The tubes are concrete-filled up to the 49th storey.

The perimeter fits in a square of side 45m rotated 45º with respect to the core. Two double-storey mechanical equipment (M&E) floors are located at storeys 28 and 47 and around these storeys the larger diameter columns slope inwards (see Fig. 1) so that the sides marked 'taper' in Figure 2 retract. At the highest storeys the office floor area is significantly reduced and the perimeter follows the shape of the core. Outriggers are installed at the M&E floors to enhance the rigidity of the building frame under wind loads.

The building symmetry suggests a pair of axes, labelled A and B in the horizontal and vertical axes of Fig. 2 and which are used for reference in describing the performance measurements

Foundations

The column bases are bolted to the foundation at basement level (B1) where they sit with the core wall on a deep stiff foundation system (Broms & Lai, 1995) comprising six inner caissons founded up to 62m deep in marine (boulder) clay and connected by a 5.5m thick concrete mat, and eight exterior caissons founded up to 40m deep and linked by deep transfer beams.

Horizontal framing system and slabs

The core and columns are connected by a radiating framework of horizontal steel I-beams, with a circumferential ring of steel beams connecting the columns. The beam-column connections are moment resisting while the beam-core wall connections are simple-pinned connections. The horizontal framing system supports the permanent steel formwork (Bondek) for the 125mm-135mm deep concrete 'office slab'. The core wall contains a cast in-situ 150mm thick RC core slab with various openings for lift shafts, stair wells and service ducts.

Lifts and core wall openings and curtain wall

Within the core, the low rise lifts reach to storey 35, permitting a wide core wall opening symmetric about axis B from storey 38 upwards. High rise lift shafts are closed from storeys 3 to 33. The building perimeter is enclosed by a curtain wall system attached by brackets to the office slab.

Construction sequence

Foundation construction began in late 1991 and work on the superstructure began in early 1993, with installation of first static instruments by Shimizu in the core wall at storey 18 followed by first readings on 30th October 1993 (defined as day 0).

Construction sequence and material data provided by the contractor are summarised Fig. 3, indicating the progress of the structural elements and the mass of all construction materials above the instrumented floors. The slip-formed core wall rose fastest, followed by the core slab with progress of the columns lagging the core wall and slowing down during steelwork erection at the tapering sections. Installation of curtain wall panels started in February 1994 (by day 125). After completion of the structural system in March 1995 (by day 437), completion of the curtain wall took a further 80 days (up to day 522).

In addition to the structural mass, water storage tanks were installed at M&E floors, at storey 65 and roof level in mid June 1995 (day 592) totalling 1.5% of the total structural dead load. Interior finishing works and installations by tenants continued even up to the end of 1996 including the fitting out of the executive club occupying the top three storeys.

MANUAL STRESS/STRAIN MONITORING DURING AND AFTER CONSTRUCTION

Fig. 4 shows plan and elevation at storeys 18/19 where a total of 56 stress and strain gauges, with built-in thermocouples were installed by the contractor. The instruments cover a 45 degree sector in the plan of the building which is fully representative due to the multiple symmetry

For the horizontal framing system, 32 strain gauges were arranged in eight sets of four at each end of both long and short horizontal floor beams, at both storey 18 and 19. The arrangement enabled determination of axial strains as well bi-axial bending, so that complete behaviour of one long and one short beam at each of storey 18 and 19 was monitored.

Within the core wall, six stress cells and two strain gauges were installed at the same height between storeys 18 and 19 in one side of the octagonal wall. The gauges are distributed evenly along half of one wall as indicated in Fig. 4.

For the columns, two stress cells and two strain gauges were installed in the concrete of each of one large diameter column and one short diameter column. Four strain gauges were installed at 90 degree intervals around each of the two steel pipes.

Strain gauges used resistance elements attached to uni-axial elements fixed to the steel beams and columns or embedded in concrete. Stress gauges used load cells in line with vertical cylinders cast in the concrete to measure effective uni-axial stress. All gauges used temperature compensation and signal cables were terminated in panels installed in a communication shaft. The first complete set of manual readings was taken in February 1994 after columns and flooring system had reached storey 19 and thereafter at intervals of two to three weeks until July 1995. By this time the structure was essentially completed, and shortly after this half of the signal cables, presenting behaviour of three floor beams, were lost. After construction, eight readings were taken up to end of 2003.

Observations from static monitoring

Fig. 5 shows the complete set of stresses and strains recorded on the core wall between storeys 18 and 19 during constructions and Fig. 6 shows the same signals for the duration of the monitoring.

Up to end of construction, stress and strain rose together in a linear relationship as structural mass accumulated above the instrumented section. The spike in stress (gauge) CW-CS-6 shows an anomaly at day 200, about the time the curtain wall reached storey 19, when a single large and unexplained oscillation, with a period of 75 days, occurred in all stress readings. Other notable features are the maxima in stresses at day 979 (October 1996) and subsequent slow decay while the strains continued to increase. During 1996 the building was fitted out and began to be occupied explaining the increases, while creep and load shedding to the columns seems to explain the change in relationship of stress and strain, which is shown in Fig. 7, with the same legend for stress signals as in Figs. 5,6.

For both columns monitored (small and large diameter) stresses and strains continued to increase as core wall stresses decreased, supporting the theory of load redistribution. Fig. 8 shows the linear stress/strain relationship for concrete in the large column, as well as the relationship with strain in the steel pipe. While concrete and steel strains in a column exhibit linear relationships in all combinations, gradients are not unity and steel strains tended to rise faster than concrete strains.

Fig. 9 shows bending moments at ends of one of the beams. Bending moment is hogging at the column (where the beam is fixed) and is not zero but sagging at the core wall where there is a nominal pinned connection. Using a simplistic calculation on the basis of the beam bending without end rotation and increments of bending moment of 50kNm since end of construction suggests relative settlement of the core by 7.5mm, in line with the 100με difference between core and column strains observed during that period. Surprisingly, limited data for other beams (before

damage to the signal cables) show that at the columns the long beams have larger bending moments than the short beams.

The data cover only a one-eighth section of the building plan and it may not be reasonable to extrapolate to the whole plan, but if the data are representative then in early 1997 the core wall sustained a dead load of 380MN, with the columns taking 120MN in the concrete and 145MN in the steel, totaling more than the estimated 520MN based on the supplied estimate of non-structural mass. By 2003 the core wall load had apparently dropped by 13% in the same section.

TRACKING NATURAL FREQUENCIES DURING CONSTRUCTION AND AMBIENT VIBRATION SURVEY

By mid June 1994 a portable acceleration recorder was available and was used to record horizontal accelerations in A and B axes aligned with the low-rise and high-rise lift lobbies respectively. These are horizontal and vertical axes, as labeled in Fig. 2. Pairs of first mode frequency values were obtained from curve fitting to auto-spectra of the signals showing doubling of both periods up to the point at the end of 1995 when a formal modal survey (Brownjohn et al., 1998) was undertaken, between 20th November and 1st December 1995. The modal survey data were subsequently used for validating a finite element model constructed using SAP2000 (Brownjohn et al., 2000).

The variation during construction of period for fundamental modes in each direction i.e. A1 and B1 is shown in Fig. 10 together with mass (above storey 18) and construction progress, as indicated by highest storey of completed office slab. It is clear that while B develops into the stiffer of the two directions, frequencies were originally identical. The difference would be due to the changing arrangement of the core wall as higher storeys. From the final stages of construction

there was no evidence that the curtain wall affected either stiffness or damping characteristics of the structure.

The procedure for the modal ambient vibration survey (AVS) was the standard technique of the era now commonly referred to as 'peak picking'. Peaks in auto power spectra were taken (with a little judgment) to indicate a mode, and mode frequency and damping were estimated by fitting curves of single degree of freedom oscillator response functions around these peaks for many data sets. Comparison of phase angles and amplitudes of cross power spectra at the estimated modal frequencies provided estimates of operating deflection shapes (ODS). For low damping and well separated modes, ODS are adequately accurate estimates of mode shape. Mode frequencies and damping ratios have since been checked using more sophisticated techniques (Brownjohn, 2003).

To map out mode shapes with good resolution over the height of the building using only four accelerometers required keeping a reference accelerometer in the same location (one corner of storey 65 close to the core wall) while moving three 'rover' accelerometers to different locations throughout the height of the building. For each measurement signals were sampled at 15Hz for as long as possible while keeping accelerometers aligned in one direction, before rotating them all by 90°, measuring again, then shifting the three accelerometers to new floors. The sequence was repeated over several days to map modal ordinates throughout the whole building in A and B directions with respect to a vertical line through the height of the building at the corner of the core wall (location 1). Mode shapes were normalized to unity and zero phase at the reference location.

As the measurements progressed it became clear that modes did not divide exactly into A and B directions, rather that the principal axes of movement were rotated unknown angles with respect to these obvious symmetry axes. Also, there was evidence of significant torsional response even in the translational modes, so a set of four measurements was made, one at each of storeys 18, 32,

46 and 65, to identify the unknown angles and mode shapes in horizontal planes using an arrangement of the set of four accelerometers.

A complete set of twelve modes was identified, numbered A1 through A4 for modes closest to the A direction, B1 through B4 for modes aligned closest to B direction and T1 through T4 for modes of almost pure torsion. Modes A1, A2, A3 and T1 are illustrated in Fig. 11 together with an auto-spectrum of A-direction storey 65 acceleration obtained during the AVS. The modal parameter estimates are given in Table 1 from the AVS as well as from a period in early 1997 when the building was occupied.

The static monitoring program up to late 1996 is described in a full internal report (Brownjohn & Pan, 1996) together with details of the natural frequency tracking and AVS procedure.

ACCELERATION RECORDING SYSTEM

Long term monitoring of Republic Plaza followed experience with a wind and acceleration response recording system installedin a 26 storey apartment block in Singapore (Brownjohn & Ang, 1998) that operated until the end of 1995, when the equipment was removed for use in the Republic Plaza AVS.

From the apartment block study and other sources (e.g. Pan, 1995), there was strong evidence that earthquakes occurring in Indonesia were inducing significant but (so far) non-destructive vibration response in residential structures in Singapore. The apartment block monitoring system was not able to record the building foundation response and the ground motions could only be inferred from the recorded rooftop response. The Republic Plaza study provided the opportunity to provide for a foundation level acceleration measurements by installing cables during construction. Thus it was intended that over the long term, synchronous roof and foundation level acceleration recordings could be used to capture ground motions at the building as well as their effects.

Hence, from October 1996 until January 2005, acceleration signals were recorded and analysed with a few breaks due to data download, hardware faults or maintenance. In the initial installation until January 1997, while two more accelerometers were procured, two accelerometers were placed in a telecoms riser cabinet in the storey 65 M&E storey. There were connected to the acquisition system used for the AVS and left recording continuously for two weeks as part of the learning experience to track the levels of normal response. During that period the response due to a strong (Ms6.3) and relatively close (epicentral distance 700km) Indonesian earthquake was recorded, being the first time series recording of building structural response in Singapore.

In January 1997, the two accelerometers were attached to the corner of the core wall (location 1 in Fig. 2) at storey 65 and an additional pair was installed in the exact same location directly below, in the basement (B1). The four signals were supplied by the pre-installed cables to a four channel signal conditioner providing, low pass filter, accelerometer offset adjust and amplification.

Honeywell QA-700 servo accelerometers were used; these units have low noise threshold (around 1 micro-g), and are immune to interference over long lengths of signal cable.

Acceleration signals were sampled in frames of 4096 samples acquired either at 7.5Hz or 8Hz. In fact the data were over-sampled in short frames and decimated in order to benefit from the sharper cut off characteristic of digital filters in the acquisition software that was written in FORTRAN to drive a 12-bit AT-bus analog to digital converter card and process the data. The system used double-buffering with one buffer being filled while data from the other were processed, involving calculation of FFTs and various statistical properties of the signals. Hence for every frame of approximately 9 minutes duration a set of parameters describing mean, variance and narrow band root mean square (RMS) corresponding to known vibration modes were stored.

In addition, when trigger conditions were met such that the signal contained interesting features, the frame of time series was saved to a compact binary file together with trigger indicators. Acceleration trigger conditions, applying only to storey 65 accelerometers, included strong broadband response as well as response in specific lower vibration modes. Such response was determined by computing RMS acceleration in narrow frequency bands around the modal frequencies.

Trigger levels were set relative to a moving average of response so that weak signals could be captured against the weaker background of building response to both wind and internal machinery at night. After a short learning period it was observed that while wind generated strongest acceleration response in the first mode (A1, B1), distant earthquakes invariably caused the strongest response in the second mode (A2, B2). Hence triggering on second mode RMS was found to be very effective for capturing local effects of distant earthquakes (Brownjohn & Pan, 2001)

From almost five years of data up to July 2001, daily maximum RMS acceleration values due to wind and excluding known earthquake response were obtained and used to produce the Gumbel plots of Fig. 12. These plots confirm the relative weakness of A-direction and that even allowing for statistical peak factors (ratio of peak to RMS) around 3.0 for a ten minute record, the serviceability criteria of 98mm/sec^2 (0.01g) established for the building are higher than the acceleration response expected for a 50-year return period.

Even the strongest ever recorded response to wind, with 15mm/sec^2 peak amplitude, is dwarfed by the acceleration response due to the magnitude 8.0 Bengulu earthquake that occurred in June 2000. Apart from the December 2004 Aceh earthquake, this was the strongest earthquake in the region during the monitoring and it generated the strongest recorded signals reaching (and possibly exceeding) the system dynamic range of 50mm/sec^2. A summary of characteristics and effects of recorded tremors up to 2000 is given by Brownjohn & Pan (2001).

The AVS was too short to identify reliably building response at the very low floors, but from an ensemble of large amplitude response time histories collected during strong winds it was possible to identify accurately the mode shape ordinates at basement. Normalised with respect to unity at the roof, for the first three modes these values are 0.006, -0.010 and 0.016. As this shows the

foundation to be very rigid, it was argued that the basement performance is a good representation of local ground movement hence, with the sensitive second mode trigger, the building works as a sensitive seismometer.

With ten years of acceleration response data it is useful to examine variations of natural frequencies as some researchers have considered them to be directly or indirectly useful as damage-sensitive parameters for structural health monitoring. For translational modes A2, B2, A3 and B3 the frequencies drop on average 0.65% per annum; Fig. 13 shows the change for modes A3 and B3. There is considerable scatter, and Fig. 14 suggests the cause from 16 days of continuous recording. The bars indicate by height (z-axis, labeled 'msv') and lightness of shading the strength of a mode identified at a time and frequency. The eigensystem realization algorithm was used, and in order to judge the repeatability of the estimates, different numbers of lags (data samples) were used to form Hankel matrix blocks from the cross-covariance functions constructed from 1Hz re-sampled time series. The two solid lines plot, with arbitrary scaling, variations of mode A1 RMS amplitude (lower line) and ambient temperature value (upper line). The ambient temperature variations would certainly differ from the structure core temperatures but there is a clear diurnal effect and no convincing effect of mode amplitude.

WIND RECORDING SYSTEM

To study the relationship between wind speed and acceleration response, a pair of three-component (UVW) propeller anemometers was added to the system in April 1997. Placement of the anemometers was complicated by height restrictions and rooftop machinery such as the gondola crane, so the only available option was to install anemometers 1.5m above the parapet corners with lightning protection up to 1.8m. While far from ideal, by placing one anemometer on each of the east and west corners of the building it was hoped to capture the strength of the

monsoon winds from the northeast direction as well as the storms and squalls predominantly from the west. Subsequent studies using computational fluid dynamics and a wind tunnel model (Tng et al., 2000) demonstrated that the placement resulted in overestimation of wind speeds by around 10% mainly due to the vertical (upwards) component.

By late 1997 individual axes in the anemometers had begun to fail, with suspicion of lightning damage. Both units were replaced by sonic anemometers in February 2000 which worked correctly until October 2000 when one unit failed, followed by the second in December. One sonic was repaired in late 2003. Hence good quality wind signals from both anemometers were available for relatively short periods i.e. most of 1997 and most of 2000, but adequate quantities of wind data were available for studying loading mechanisms.

From these data, Gumbel plots shown in Fig. 15 were produced. Even allowing for deficiencies in the anemometer location, the plots indicate that the strongest winds in Singapore are the storms and squalls from the west, while north-easterly winds, typical of monsoon winds are relatively weak even if more persistent. The 50-year return period winds are rather low and a study of seismic and wind loading code provisions for this building (Brownjohn, 2005) has shown that built in resistance for accidental eccentricity provides adequate cover for lateral wind loads.

RECOVERY OF DISPLACEMENTS FROM ACCELEROMETER SIGNALS

An original purpose of the monitoring was to infer the wind loading via the observed effects. Following the demonstrated capability of the system to detect and record tremors from around the region, tremor detection became a priority, but with development of a Singapore-specific wind loading code at the time of the project, Republic Plaza provided an opportunity to calibrate the candidate code provisions.

As with a UK exercise (Littler & Ellis, 1990) in which an empty apartment block was used to study total static and dynamic response to wind load, Republic Plaza provided a convenient form of wind sensor. Unlike earthquakes, wind loading and its effects comprise static (mean or DC) and dynamic components, and modern wind loading codes deal with this via a dynamic magnification factor multiplying the static effect of mean wind and including the effect of both broadband turbulence and response in first-mode resonance. Calculations based on the most recent Australian wind code (AS/NZS, 2002) give a dynamic amplification factor close to two, but this depends on the character of the wind, particularly the turbulence, which for Singapore is relatively high.

The majority of wind-induced resonant response is in first mode and Fig. 16 shows how mode A1 acceleration varies with dynamic component of the wind, represented by the product of mean and standard deviation. Given mode A1 circular frequency ω_{A1}, displacement can be estimated by dividing modal acceleration by $\omega_{A1}^2 \approx 1.5$.

Apart from issues of occupant comfort during first mode sway due to wind turbulence, it is the total wind force that concerns designers. The total internal forces, primarily base shear, can be calculated if the total deflection is known, given the validated finite element model. Knowing the mass distribution of the building along with mode shape and first mode acceleration it is straightforward to find the resonant component of base shear.

The non-resonant background component can be estimated via integration of acceleration, but it is impossible to recover the contribution right down to DC.

This is because the double-integration approach is subject to contamination by instrument noise at very low frequencies, and this translates to high displacements. Fig. 17 shows B-direction displacements obtained from integration of accelerometer signals from the 2004 Bengukulu earthquake and aftershock, using a cut-off frequency of 0.02Hz. Apart from the obvious first mode contribution of storey 65 response, the two signals are identical indicating a rigid body motion of the entire building. Using a lower cut-off frequency in the integration introduces low frequency ripples in different channels at different times due to noise, so the result also fixes a reasonable lower cut-off frequency for general application of the integration process and effectively rules it out for the very slow quasi-static wind-induced response.

An alternative method assumes that, as in a cantilever, deflection is accompanied by some degree of rotation which would be evident as a shift in mean acceleration of the horizontally aligned DC accelerometers due to sensing a component of gravity. Since the accelerometers operate down to 0Hz (DC) a rotation of the accelerometer by a small angle α is sensed as a (mean) static acceleration. This presupposes reliable acceleration signals yet there is certainly a strong influence of thermal drift. In late 2001 accelerometers were replaced with units having 'thermal modelling' for correction of thermal bias via a temperature signal from the accelerometer. Even with the conditioning system having high thermal stability and after correction for thermal bias there was evidence of thermal drift effects in the data, so that the slowly varying mean acceleration could not be relied on to recover deflection. Given the issues of noise and thermal drift, a more reliable means to recover deflections was sought.

GLOBAL POSITIONING SYSTEM

With a proven track record in recording dynamic structural deflections in suspension bridges (Ashkenazi & Roberts, 1997), the global positioning system (GPS) offered possibilities of

absolute position measurement for resonant, dynamic non-resonant and static responses. Hence, in 1999 the existing system was upgraded to include a dual-rover real time kinematic (RTK) differential GPS system.

For a single receiver (rover), estimates of position are subject to errors that depend on modification to signal transit time due to atmospheric and other effects. If a fixed base or reference (base) station is located nearby, given the known fixed location, the 'differential' errors can be identified and used to adjust the rover position estimate. When the errors are transmitted from base station to rover and incorporated in rover position estimates in real time, position fixes accurate to the order of a centimeter or better are possible. RTK operation uses software embedded in the receivers to correct for the errors, but if the original 'raw' data are saved for each receiver, subsequent post-processing can provide position fixes taking advantage of more sophisticated off-line processing, potentially providing more accurate data. More recent GPS capabilities allow for direct supply of corrections without the need for a base station.

The system at Republic Plaza was designed to operate in both RTK and off-line post-processing modes. RTK solutions output at 1Hz for each sample as text data (referred to as NMEA) were converted to analog signals and supplied to and recorded by additional channels on the existing data acquisition system. While it is possible to use GPS at higher sample rates to capture higher mode dynamic response (Li et al., 2004), resolvable response at second mode frequencies would have required seismic accelerations likely to cause structural damage to the building. As this was a negligible possibility, a slow sample rate was used to minimize data transmission and storage problems.

The event trigger generated by the analog system was connected to the GPS recorder to trigger saving of the contents of a ring-buffer of raw data acquired by the computer controlling the two

rovers. 30 minute pre- and post-triggers were used; a long pre-trigger was required by the post-processing for a good solution at the time of the trigger. Due to slow data transmission rate from the base station, one hour of data sampled at 1Hz required two hours to transmit. The post processing itself was also a slow process and RTK data had the advantage of ready availability and synchronization with the analog signals.

The rover antennae were positioned close to the anemometers, flush with the level of the parapet, and the base station was located on a low rise building 10km away. As with the anemometers, the antenna configuration was not ideal; it was not possible to use larger, more expensive and more accurate choke ring antennae and the low level mounting was intended to reduce likelihood of lightning strikes (one antenna had to be replaced).

The initial GPS data appeared very noisy and not to correlate with other signals, hence it was difficult to believe what the data represented. Validating the GPS data was a major issue, as the signals were subject to various forms of error such as possible multi-path, cycle-slip, random noise and systematic noise. Also, the total movements of the building, expected to be of the order of +/-0.1m, were expected to comprise components of dynamic and static response to wind as well as static response to temperature changes in and around the building. Direct evidence that the system was working was obtained first by physically moving the antenna during a recording and secondly by studying the signal during strong winds and or earthquakes generating first mode deflections at least 1cm amplitude.

Two periods in early 2003 and early 2004 generated useful GPS data during the relatively strong seasonal monsoon winds, and several months of RTK data were recorded continuously. However, much of it was affected by periodic over-ranging which occurred at slightly irregular intervals of approximately 30 minutes. A good example of typical data is shown in Fig. 18, representing a

storm generating first mode response with amplitude 10mm/sec^2. The RTK data shown were corroborated by the second rover and indicate a steady drift of the building with a kind of ratcheting, together with dynamic response.

Surprisingly, the best validation of the GPS system performance was provided by data recorded during the December 26th 2004 earthquake off the coast of Sumatra, Indonesia. Fig. 19 shows storey 65 displacement in A-direction relative to basement recovered from the acceleration signals, and also displacements obtained directly using RTK from the two rovers. The RTK displacements are the A-direction component of the vector sum of Eastings and Northings, band-pass filtered between 0.1Hz and 0.3Hz.

While storey 65 acceleration can be integrated to show the type of large low frequency displacements visible Fig. 17, RTK does not, because the displacements were relative to the base station, which moved to the same extent as the ground at Republic Plaza.

OBSERVATIONS ON STRUCTURAL AND LOADING MECHANISMS

Even after ten years of monitoring there are still mysteries about the building performance. For example the static deflections due to temperature were not determined and presented a problem as temperature affected the instrumentation as well as the structure movements. Given that cladding should act as a non-structural insulator it is more likely that thermal cycling is due to air conditioning operation. The way the building responds to mean and slowly varying winds is still not understood; no obvious pattern of displacement response emerged.

It is clear that not only does the core wall bear the bulk of the lateral load, it also carries the majority of the vertical load, which due (apparently) to concrete creep, has gradually redistributed

to the columns, with resulting bending moments induced in the horizontal framing system. From tracking modal parameters during construction it was clear that curtain wall did not contribute either to stiffness or damping.

From study of dynamic response at the basement, the caisson foundation is massively stiff and must closely follow response in the area of the building. Hence the building can act as an excellent tremor detector, using the second mode response at roof level as a sensitive trigger. It is not absolutely certain to what extent the building basement movement represents the 'free field' motion, but the rigid body motion at long periods (20s seconds to the limit of integration at about 50 seconds) almost certainly represents a larger scale ground motion in the greater area of Singapore and evidence from the GPS supports this.

Over the ten years of monitoring, the dynamic characteristics in the lower modes have changed only slightly, with a detectable gentle downward trend in the higher mode frequencies. The effect is open to interpretation and could result from changes in mass and/or stiffness. Damping, ranging between 0.5% and 1% was identified a number of ways including the traditional circle fit method applied to transmissibility function between roof and basement, but accuracy of the estimation process was not sufficient to allow detection of possible small variations due to structural changes.

One of the biggest surprises was that, for Singapore, dynamic loads due to distant tremors are far in excess of the dynamic loads due to wind and probably also the total (dynamic+static) wind load. As the basement response spectra show a concentration of energy between 0.5Hz and 1.0Hz there are implications for the majority of tall buildings on the island having this range of first mode frequency and attracting even greater seismic loads due to the higher (first mode) participation factor.

EXPERIENCE WITH INSTRUMENTATION AND MONITORING SYSTEMS

The exercise provided good experience with designing, installing, upgrading, operating and maintaining a structural monitoring system. Such experience is relevant as simple monitoring systems evolve into structural *health* monitoring systems.

Up until early 1997, all sensors, installations and wiring had been organized by the contractor, Shimizu Corporation. After handing over, the new owners, City Developments Limited, continued to provide assistance and access. From this point, equipment was upgrades and maintained by NTU Singapore.

Both propeller and sonic anemometers were unreliable and required frequent replacement or servicing. In the end, given the mis-representation of free-stream wind velocity by anemometers constrained to be located on the parapet, and a relationship of dynamic response to wind strength established during the periods of trouble-free operation, a simple cup and vane anemometer should suffice as qualitative indication of wind strength and direction.

The data acquisition program and communication software were DOS-based, occupied little disk space and memory and so were stored on a non-volatile RAM disk with the operating parameters. Response data were stored on random-access files written to dual (RAID) hard disk drives, as early problems of disk failure and system hang resulted in loss of data.

Failures of various components at different times resulted in relatively short periods of trouble-free high quality data collection, but the accelerometers and signal conditioning system (built by

University of Bristol in 1994) were perfectly reliable for almost ten years of continuous operation and it is data from this part of the system that proved the most valuable.

The experience with GPS was below expectations. The system, which was necessarily located in the sheltered but hot area of the building roof, occasionally overheated, and leased telephone lines were noisy. There were glitches with transmission of corrections for the RTK operation and even with good correction data, issues of multi-path, cycle slip and random noise due to different transmission path mean that it was difficult to establish the quality and reliability of the signals. Difficulties were not mitigated by pushing the technology to the limit in terms of implementation and demands on resolution and accuracy. The experience shows that dynamic response down to frequencies of about 0.02Hz is capably resolved by good accelerometers yet GPS remains the only viable solution for real-time tracking of quasi-dynamic response provided building movement is at least of the order of centimetres.

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The program has been a rewarding exercise in collaboration and interaction of a spectrum of enthusiasts (too numerous to mention) in building performance assessment. The authors are particularly grateful for assistance provided now and in the past by Shimizu Corporation who also provided Fig.4, CDL, colleagues and students from NTU Singapore, SysEng (S) Pte Ltd and UNSW (Australia).

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Table 1 Modal parameters for Republic Plaza: unoccupied in 1995, occupied in 1997

A-mode	Frequency/Hz		ζ/%	B-mode	Frequency/Hz		ζ/%	T-mode	Frequency/Hz		ζ/%
	1995	1997	1997		1995	1997	1997		1995	1997	1997
A1	0.192	0.184	0.66	B1	0.201	0.194	0.70	T1	0.564	0.528	0.53
A2	0.702	0.676	0.85	B2	0.749	0.726	0.52	T2	1.341	1.258	1.25
A3	1.553	1.490	0.87	B3	1.739	1.690	0.77	T3	2.309	2.205	1.65
A4	2.486	2.403	0.74	B4	3.004	2.904	1.67	T4	3.329	3.140	1.59

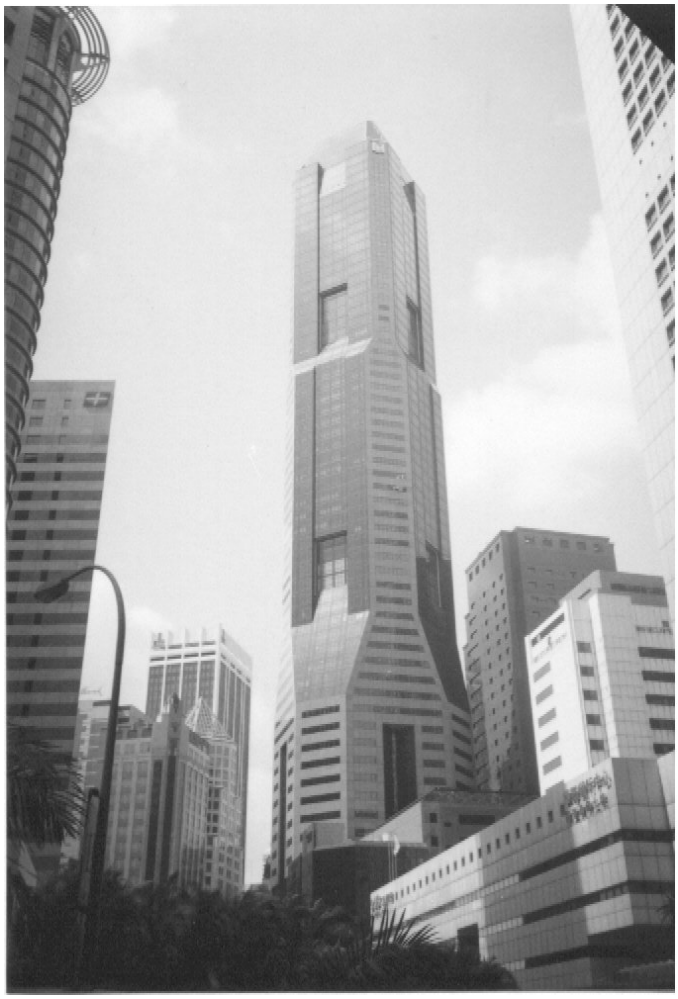


Figure 1 Perspective view of completed building

Figure 2 Typical cross-section, at storey 18.
Coordinate system for measurements is labeled A,B.

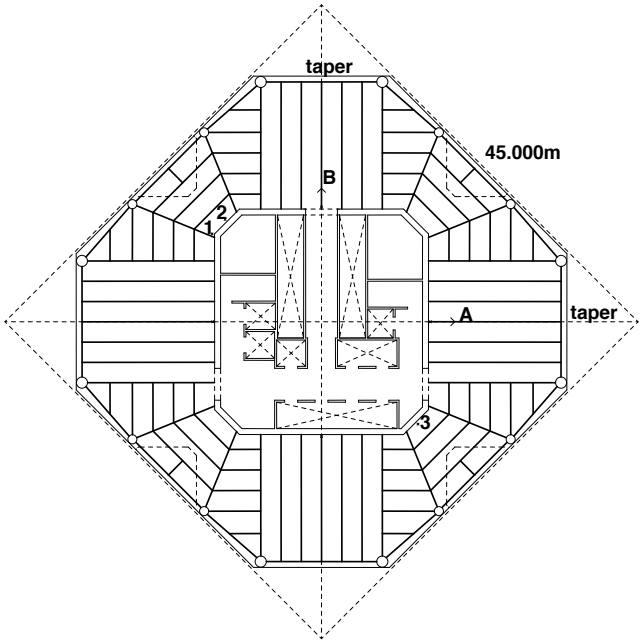


Figure 3 Construction sequence and total mass above instruments

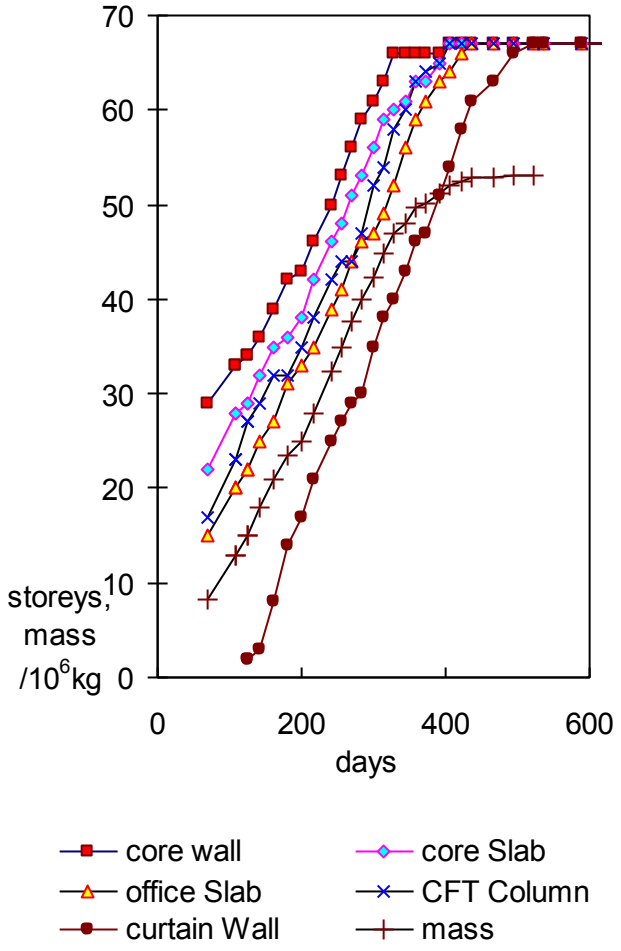


Figure 4 Arrangement of stress and strain gauges:
Strain gauges are attached to one short I-beam and one long I-beam (shown in plan in top view and elevation in bottom view) at storeys 18 and 19, at each end (core and column). Four gauges are arranged on upper and lower I-beam flanges. Six stress and two strain gauges are arranged (as shown in mid view) in core wall between floors.

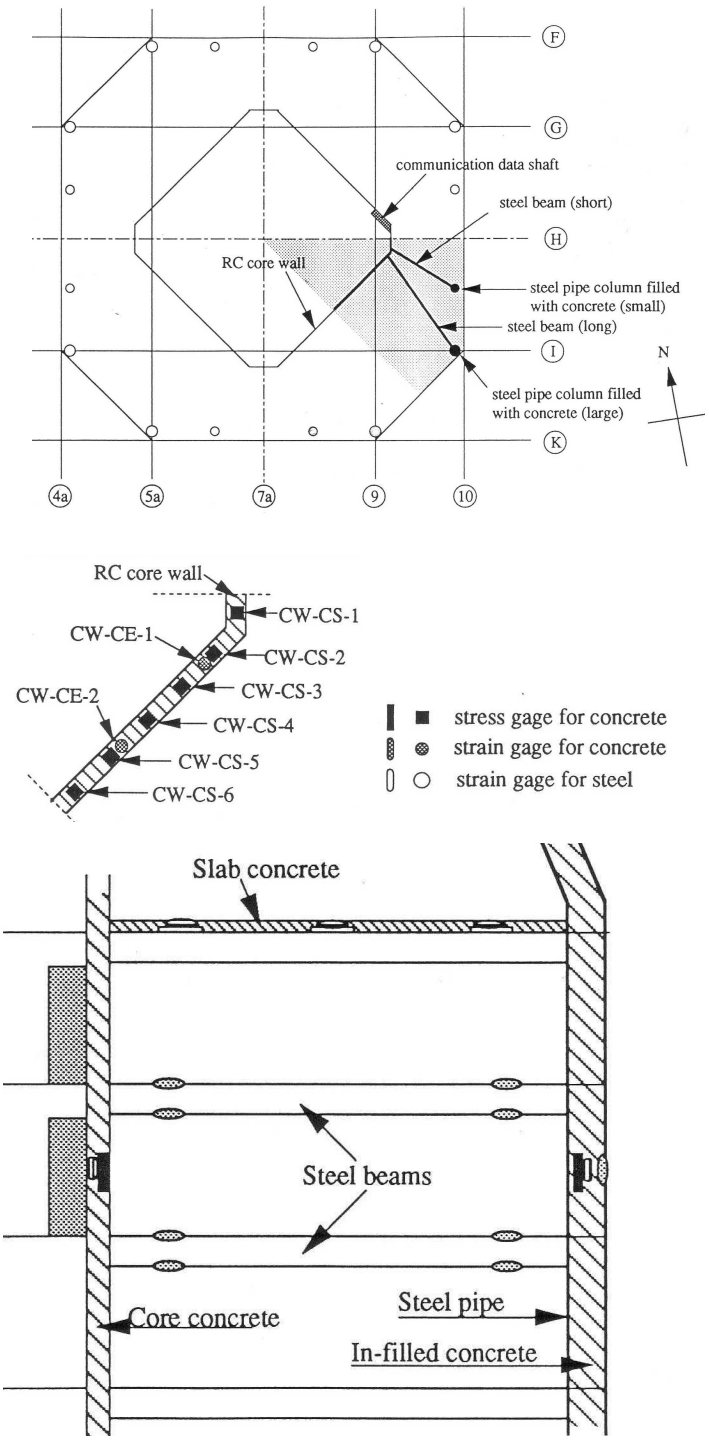


Figure 5 Core wall stresses and strains during construction. CS is concrete stress, CE is concrete strain. Instrument locations are indicated in Fig. 4.

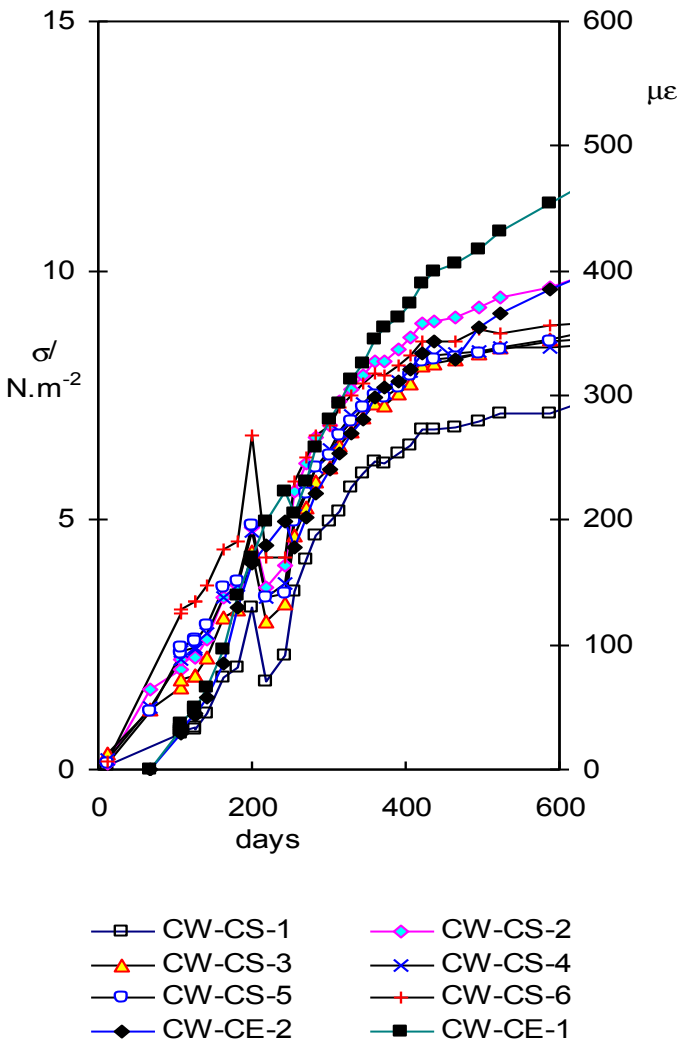


Figure 6 Core wall stresses over duration of the monitoring. CS is concrete stress, CE is concrete strain. Instrument locations are indicated in Fig. 4.

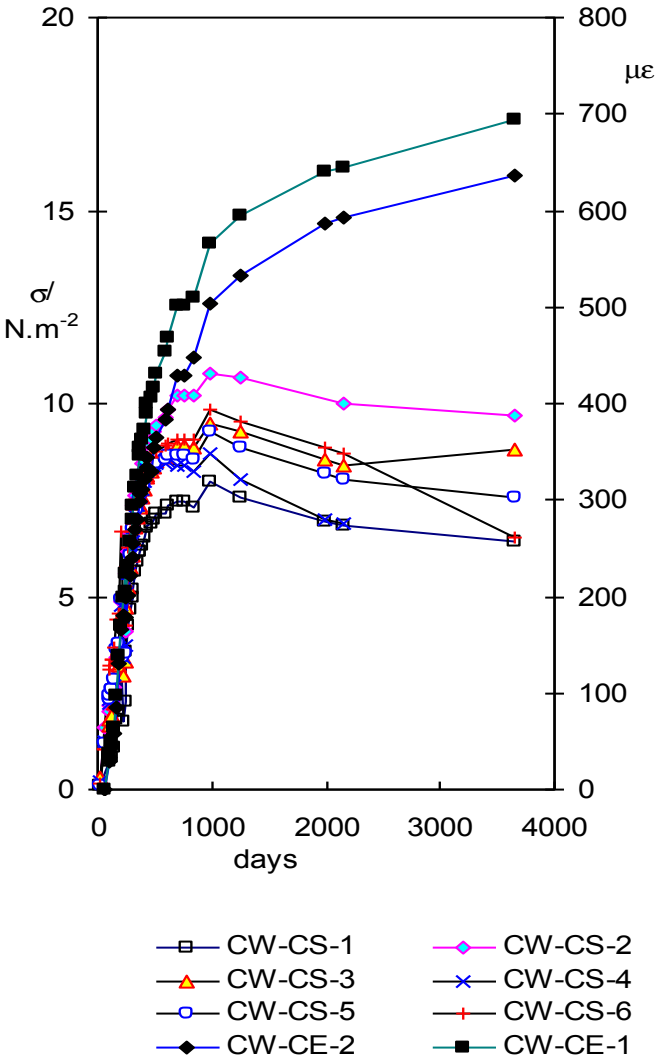
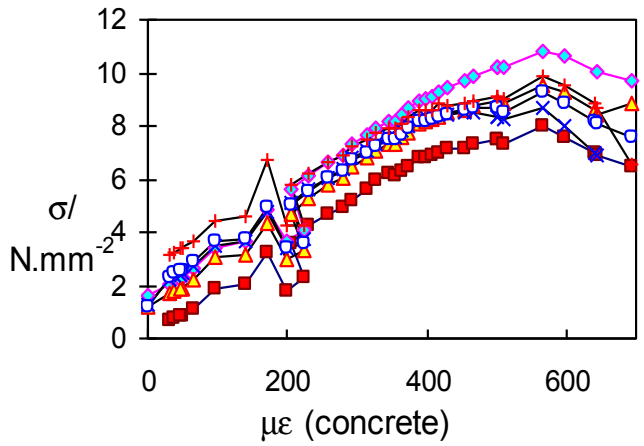


Figure 7 Stress-strain relationship in the core wall for the six stress cells.



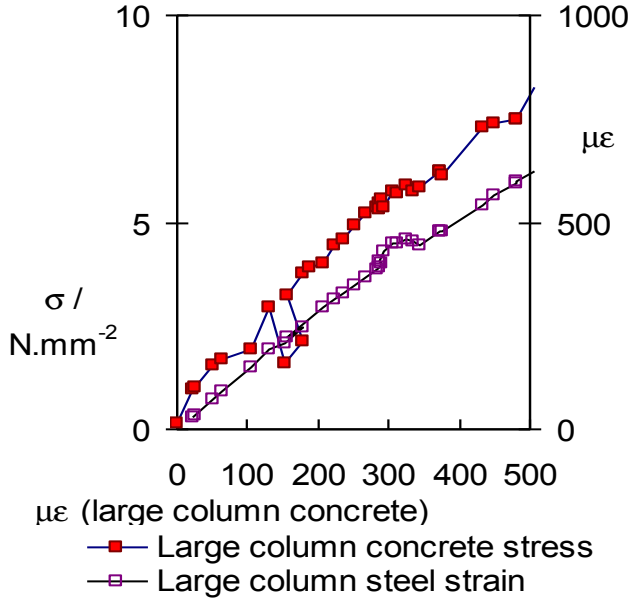


Figure 8 Stress-strain relationship in large columns

Figure 9 Bending moments in storey 19 floor I-beams
Locations of the beams are indicated in Fig. 4

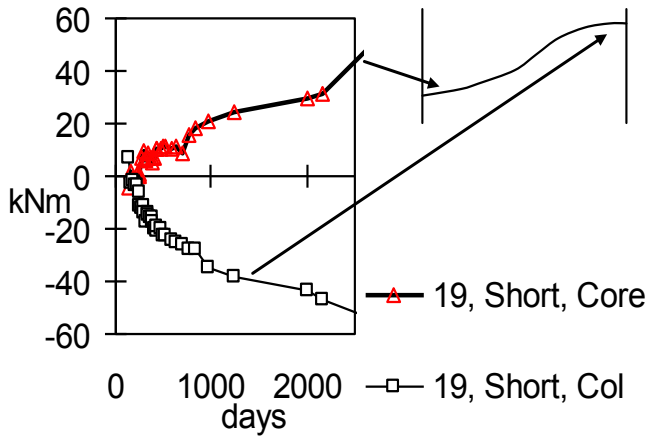


Figure 10 Variation of fundamental mode periods during construction

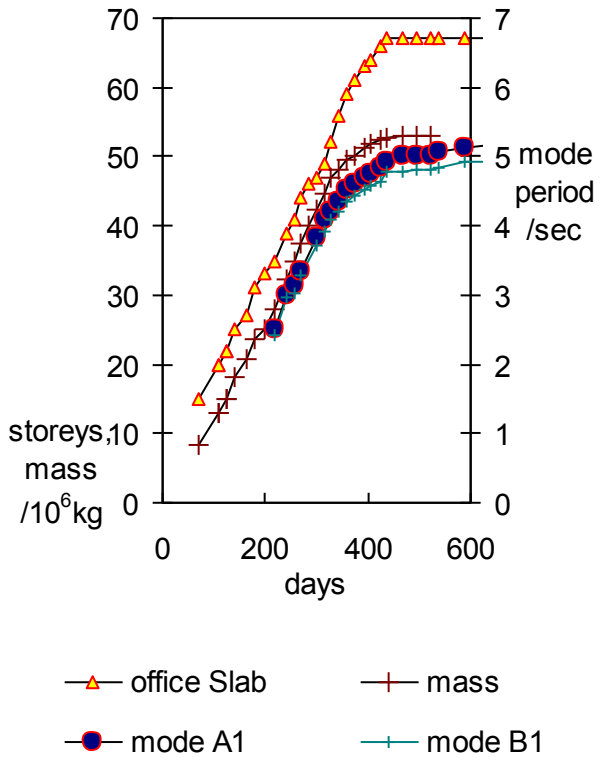


Figure 11 Vibration auto-spectra at storey 65 and measured modes

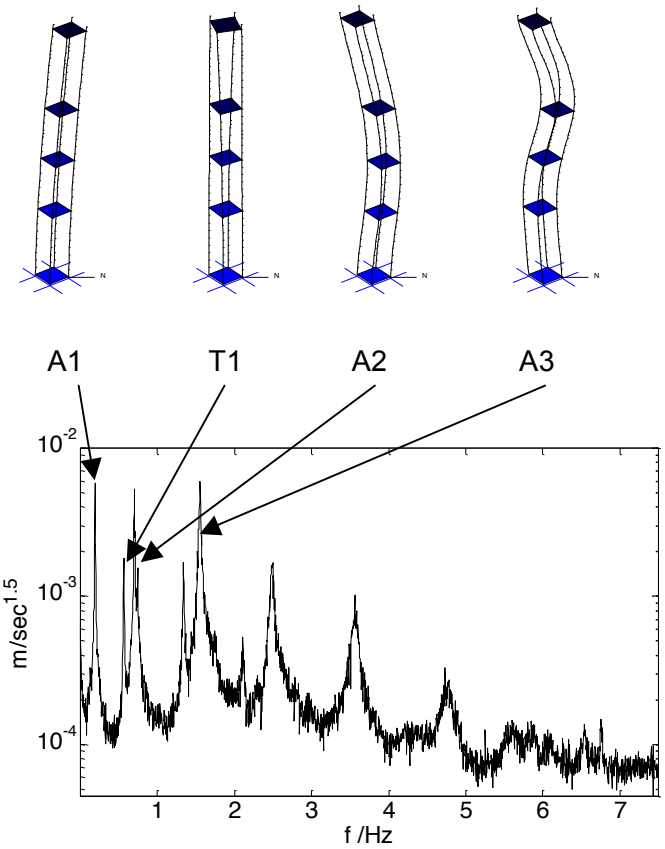


Figure 12 Gumbel plots of storey 65 first mode RMS accelerations

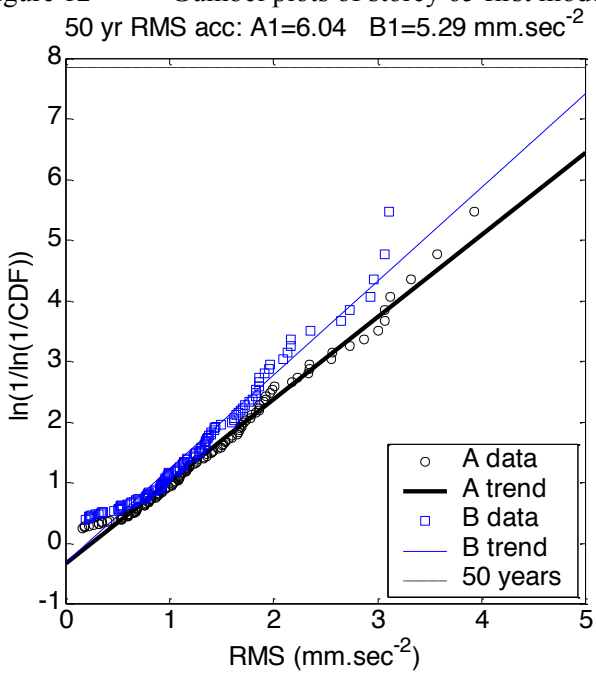


Figure 13 Long term trend of mode A3 frequencies (lower curve) and B3 frequencies (upper curve)

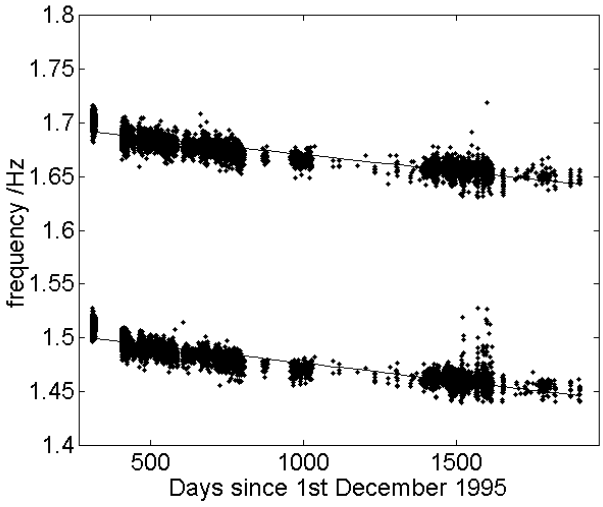


Figure 14 Short-term variation of mode A1 and B1 frequencies compared to RMS accelerations (next to time axis) and ambient temperatures (shown without scales)

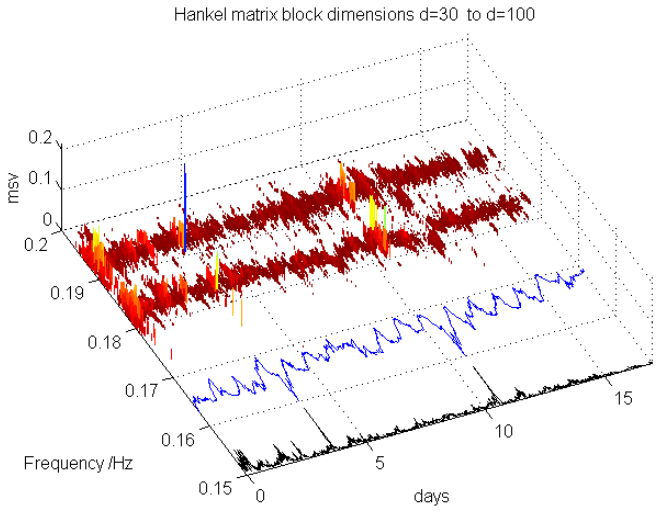


Figure 15 Gumbel plots of east and west total mean wind speeds

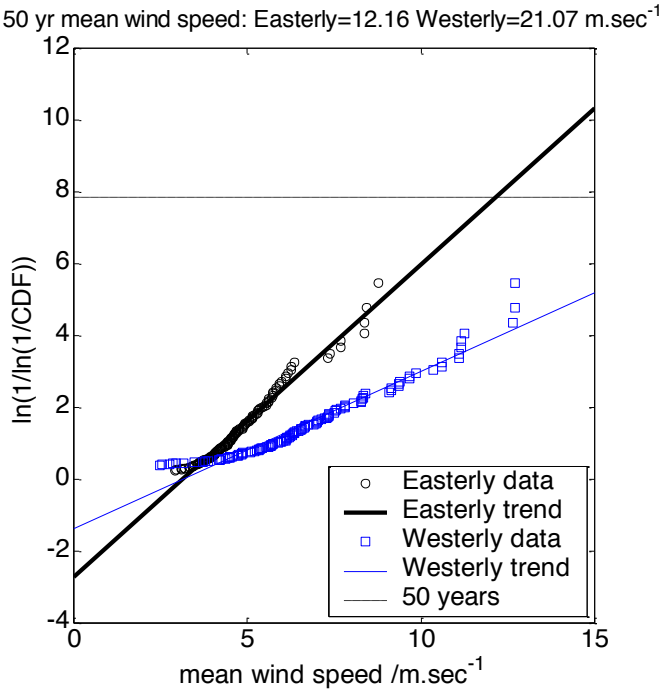


Figure 16 Relationship of mode A1 acceleration response to dynamic component of wind

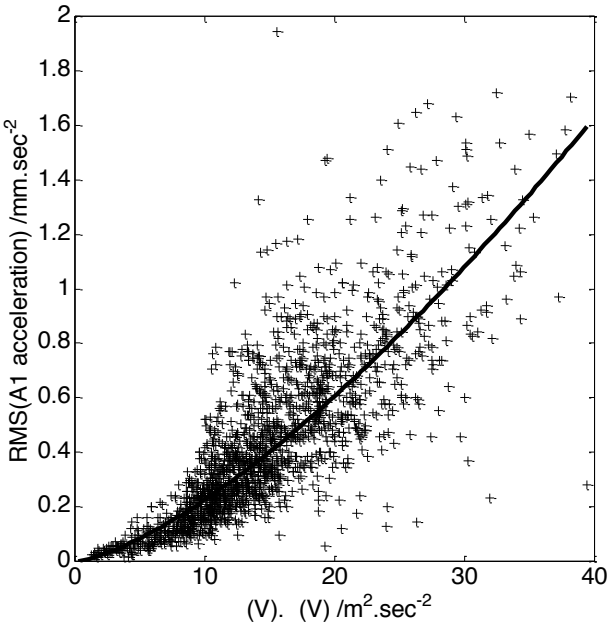


Figure 17 B-direction displacements at basement B1 and storey 65 recovered by integrating accelerations resulting from 'great earthquake' near Singapore in 2000.

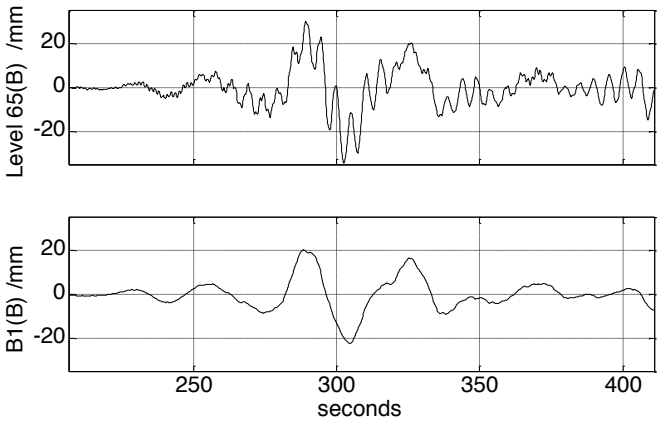
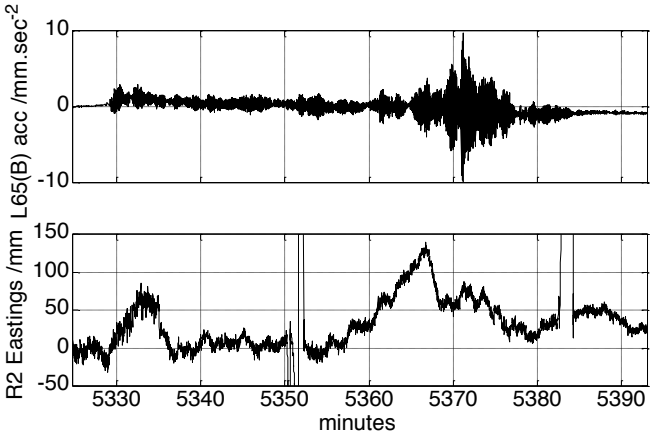


Figure 18 1Hz sampled time series of storey 65 accelerations and RTK displacements during a storm



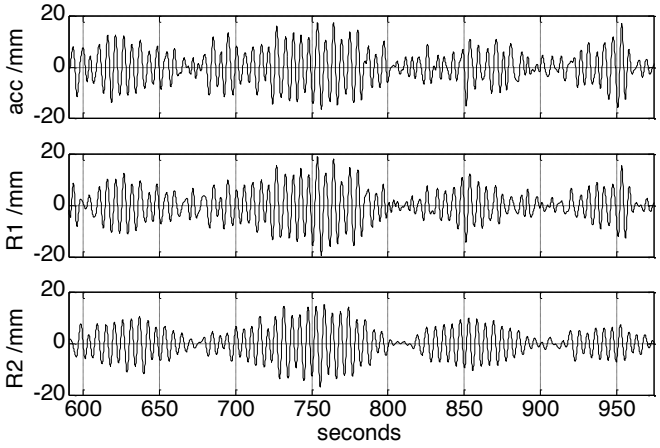


Figure 19 Evidence of GPS performance: correspondence of A-direction relative displacements recovered from accelerometer (acc) compared with RTK-GPS displacements (R1, R2) during Aceh earthquake on 26th December 2004.